GEOTECHNICAL EVALUATION REPORT

A detailed geotechnical evaluation of the Gilbert and Bennett site and of the proposed containment cell was performed by Malcolm Pirnie to address comments in the Connecticut Department of Environmental Protection (CTDEP) September 18, 1990 NOD. The principle issues raised by the NOD included:

- The adequacy of the underlying native soils, bedrock and fill materials to support the stabilized by-product and cap without compromising long term cap integrity.
- The stability of the proposed landfill slopes and need to provide slope reinforcement.
- The potential for erosion along the Norwalk River embankment and need to provide protective rip-rap.

To address these concerns, an evaluation of the geotechnical properties of the site soils was performed. This evaluation consisted of a review of site stratigraphy, interpretation of data from boring logs and supplemental analyses of 47 samples for physical properties including natural water content, grain size distribution and specific gravity. The exploration data gathered during recent investigations was reviewed to assess the affects of subsurface deviations from those interpreted at the location of the developed cross section. The stability of the proposed waste containment cell, the existing embankment along the Norwalk River ("the eastern perimeter embankment") and the proposed RCRA cap have been analyzed using the proposed RCRA cap cross-sections and final grading plan of the closure site and cross-sections of existing site conditions. Our stability analyses were performed using the simplified Bishop method, utilizing the STABL Slope Stability Computer Program developed at Purdue University[1]; results were confirmed using the Modified Swedish Method as outlined in the Army Corp of Engineers Manual EM 1110-2-1902[9]. We have also made recommendations concerning the placement of rip-rap along the embankment as a protective measure against erosion for the 100-year storm event.

RIP-RAP REQUIREMENTS:

100-year storm flow elevations, velocities and discharges were used to calculate rip-rap requirements for erosion protection along the western bank of the Norwalk River. Utilizing the National Cooperative Highway Research Program, Report No. 108, "Tentative Design Procedures for Rip-rap Lined Channels" (Revised April 1987), median rip-rap size (d_{50}) was determined to be 5 inches, placed to a 10 inch thickness over a geotextile fabric at a maximum vertical side slope of 2.5H:1V. This will require that the side slope of the eastern perimeter embankment be cut back and flattened from its approximate existing 2H:1V slope to accommodate the rip-rap. The detailed calculations and supporting documentation for this analysis are presented in Attachment E-1.

ASSUMED SOIL PROPERTIES:

Determination of soil properties to be used for the performed analyses was made using information obtained during Malcolm Pirnie's field investigation in mid-1991. Boring logs presented in Appendix C of the Site Characterization Report (Appendix D of the Closure Plan) were assessed and additional geotechnical information from laboratory investigations were analyzed. This information included analysis for natural water content, grain size distribution and specific gravity for soils underlying the proposed capping area and the existing eastern perimeter embankment (see Attachment E-2). This information was used, along with the typical site cross section (see Figure 1) to serve as the basis for our slope stability analyses. The cap stability analysis was performed on two cap cross-sections (see Figures 2a and 2b) for Alternatives 1 and 2.

Native soils underlying the contaminated sludge and sludge/soil mixtures at the Gilbert and Bennett site are predominantly well graded sands (SW), fine to coarse with loose to medium densities as determined from Standard Penetration Test (SPT) blow count results obtained during site explorations. Traces of silt, clay and gravel were detected in certain areas. Blow counts for these sands were determined based on 12 inches of penetration of a 2-inch outer diameter split spoon sampler, using a 140 lb. hammer, falling 30 inches. Blow counts generally ranged from 5 to 30 blows per foot. A weaker sand layer averaging 5 feet thick was encountered at depths of approximately 10 to 20 feet below the existing ground surface. Blow counts for these layers generally ranged from 5 to 10 blows per foot. Competent bedrock was generally encountered within 10 feet of the ground surface at the northwest border of the site (MW-101D). It was generally found to dip downward towards the east to a depth of 64 feet at MW-104D.

Soil properties were assessed based upon the available geotechnical information. Assumed conditions are summarized as follows:[3,8].

- Capping (general fill and topsoil) and impounded waste material (fine to coarse sands stabilized with a fixative agent and either crushed and compacted or solidified) were assumed to have equal strengths for the overall impoundment stability analysis. Relatively conservative strength values were utilized when analyzing the stability of the waste (friction angle, ϕ, was varied from 20° to 30°, moist unit weight, γ_t, was assumed as 110 pounds per cubic foot, (pcf) and saturated unit weight was assumed as 115 pcf (Layers 1 and 2)).

- Undisturbed native bearing soils with higher blow count values (medium to coarse sands) were assumed to be stronger than waste materials: $\gamma = 30° - 32°$, $\gamma_t = 110$ pcf (Layers 5,6 and 7).
- Undisturbed native bearing soils with lower blow count values (fine to medium sands) were assumed to have the following properties: $\phi = 26° - 29°$, $\gamma_t = 110$ pcf (Layers 3 and 4).

SUMMARY OF ANALYSES STUDIED:

Stability analyses were performed based upon the potential failure of the impoundment structure as a whole, incorporating cap, impounded waste material and underlying native materials in the analyses. Analyses were based upon the available geotechnical data including soil classification, blow count data and the proposed cell configuration. A bearing capacity failure or shear failure of the underlying materials is not expected to occur due to the adequate confinement existing between the eastern perimeter embankment and the shallow bedrock along the westerly portion of the cell. Modes of failure were assessed with regard to slope stability with failures along assumed shear planes analyzed using conventional limit equilibrium analysis. The following cases were analyzed for potential failure within the waste and cap layers as well as within the underlying native materials:

Containment Cell Stability

- <u>Cases A and B</u> - Shallow failure surfaces which represent potential failure prior to cap installation and potential failures within the cap. (Note that separate analyses for cap stability alone are presented later in this report.)
- <u>Cases C and D</u> - The impounded waste and cap acting as one unit, with failure at the toe of the impoundment cell and failure at the toe of the eastern perimeter embankment.

Eastern Perimeter Embankment Stability

- <u>Cases E and F</u> - The eastern perimeter embankment with failure at the toe of the embankment.

See Figure 3 for schematic representations of geometries analyzed.

Our analyses have been performed to determine the strength of the stabilized waste or in-situ soils (represented by internal angle of friction - ϕ) necessary to provide for stable slopes. A factor of safety (FS) of 1.5 was used as criteria for long term stability.

STABILITY ANALYSES OF CONTAINMENT CELL:

Stability of 4H:1V Slope

Analyses were performed for the containment cell with an assumed 4 Horizontal to 1 Vertical (4H:1V) slope. The existing eastern perimeter embankment (currently at a existing 2H:1V to 2.5H:1V slope) will be at the toe of the containment cell along the Norwalk River. As part of our analysis we looked at three different eastern perimeter embankment configurations (i.e. embankment slopes of 2H:1V, 2.5H:1V and 3H:1V). It was found that the results of Cases A - D were unaffected by the geometry of the eastern perimeter embankment. Cases A - D for Geometry 1 (see impoundment at 4H:1V and perimeter embankment at 2H:1V - Figure 4) represent the potential failure of the waste prior to cap installation. It is calculated that the strength of the embankment materials, represented by the internal friction angle, ϕ, for the waste must be $\geq 21°$ for the required factor of safety (FS) of 1.5. Calculated factors of safety with varied friction angle ϕ, representing impoundment cell material strength are summarized on Figure 5. The soil properties assumed for the analyses are summarized in Table 1.

Stability of 3H:1V Slope

Additional stability analyses were also performed on the impoundment cell and the proposed capping system based on an impoundment side slope of 3H:1V. These analyses were performed based upon CTDEP comments indicating that additional soils (which pass TCLP but still have measured levels of contaminants) may need to be excavated and placed within the containment cell, thereby potentially increasing the required storage volume. As for the 4H:1V slope, Cases A - D for Geometry 4 (impoundment at 3H:1V and perimeter embankment at 3H:1V as shown on Figure 3) represent potentially deeper failure surfaces within the impoundment cell (critical failure surfaces analyzed are shown on Figure 6). The ϕ for the waste must be $\geq 26°$ for a required factor of safety of 1.5 prior to cap installation (see Figure 5 for a summary of analyses performed). The assumed soil properties are summarized in Table 2.

STABILITY ANALYSIS OF EASTERN PERIMETER EMBANKMENT

Analyses for the eastern perimeter embankment, Cases E and F, were performed for: the approximate existing slope of 2H:1V with no rip-rap on the embankment; a 2.5H:1V slope (the maximum vertical slope proposed for rip-rap stability); and a 3H:1V slope. Based on analyses performed, critical failure surfaces are shown on Figures 4 (2H:1V slope), 7 (2.5H:1V slope) and 8 (3H:1V slope). Critical failure surfaces were found to exist for Case F as shown on the Figures. Surcharge was considered in Case F and it was found, for shallow surfaces analyzed, results were independent of traffic surcharge. The following factors of safety were calculated for Case F:

Eastern Perimeter Embankment Side Slope	Calculated FS
2H:1V	1.16
2.5H:1V	1.32
3H:1V	1.50

The assumed soil properties for the above analysis are summarized in Tables 1, 3 and 4 respectively. Parametric analyses were performed to determine the internal angles of friction necessary for factors of safety of 1.5. Based on these analyses it was calculated that the existing embankment at a 2H:1V slope must have a $\phi \geq 35°$ for a required FS of 1.5 and for a 2.5H:1V slope, the ϕ must be $\geq 32°$. Based on the available geotechnical information, the ϕ for the embankment material (Layer 6) and the underlying material (Layer 4) may be as low as 30° and 28° respectively.

Assuming the embankment material has a ϕ of 30° and an underlying material ϕ of 28°, the slope must be no steeper than a 3H:1V to satisfy the required FS of 1.5. However, if the presence of the "weak" underlying material Layer 3 (assumed $\phi = 26°$) is more shallow and extends further to the river than assumed during this analysis, the slope at 3H:1V may not be stable. Therefore, it is recommended that for a 3H:1V slope, the Contractor be required to perform confirmatory borings as an initial step in the construction process to assess the reasonableness of our assumptions. If space and construction requirements necessitate a slope steeper than 3H:1V, additional explorations are recommended as a part of the predesign activities.

Traffic or surcharge loads alongside the embankment will tend to make potentially deeper failure surfaces more critical. The affect of surcharge has been analyzed for the various embankment configurations under Case E. Magnitude of surcharge and proximity of surcharge to the top of embankment are critical factors in the analysis. Proximity of surcharge to the embankment will depend on the embankment side slope. Flatter side slopes, although tending to make the embankment more stable without surcharge; provide less working room at the top of the embankment resulting in the potential for loading closer to the slope. The net result is that the same surcharge load will make the flatter slopes less stable since they are applied closer to the slope. Current analyses show that for a 2.5H:1V slope, the allowable surcharge is 1840 psf, maintaining an offset 12 feet from top of slope. At a 3H:1V slope, the allowable surcharge is 1600 psf, maintaining an offset 7 feet from the top of slope (see Figure 3). Due to the temporary nature of the surcharge loading, analyses were based on an allowable FS of 1.3. Assumed geometries are shown on Figure 3. A summary of our findings is represented on Figure 9.

Although construction equipment may yield heavy wheel loads, loads tend to be distributed more uniformly as distance of application of the load increases away from the slope. The loads shown (1840 psf and 1600 psf) may be conservatively considered as representative of axle loads of 9 ton/axle and 8 ton/axle for heavy equipment with approximate 10 ft. wheel spans. In any case, as a part of the work, the contractor should be required to submit, for approval, proposed construction equipment and associated contact ground pressures prior to mobilization for the work.

STABILITY ANALYSES OF CAP:

The analyses for cap stability were performed by the Modified Swedish Method of Analysis and checked using a method suggested by Koerner and Hwu, 1991[5]. The method of Koerner and Hwu is a numerical solution of a wedge method of analysis as developed by the U.S. Army Corps of Engineers. For a 4H: 1V slope, factors of safety were calculated for various friction angles utilizing both stability methods of analysis (See Figure 10). Both methods provided similar results.

As a part of the analyses, the minimum interface friction angle was varied, the slope was varied from 4H:1V to 3H:1V, the slope length was varied from 25 ft. to 100 ft., and the strength (friction angle) of the cover soil was varied. Findings indicated the following:

- The stability of the cap is governed by the interface friction angle, δ, between the cap constituents.
- The cap is less stable as the slope steepens.
- The cap is less stable as failure surface length increases.
- Varying the friction angle of the cover soil, (ϕ_c), above the sliding plane interface has little affect on the factor of safety.

The results of analyses performed are summarized on Figure 11.

Two cap cross section alternatives, Alternative 1 (Claymax, PVC, drainage net/filter fabric combination) and Alternative 2 (Bentomat, Textured HDPE, Geocomposite drainage net), as shown in Figures 2a and 2b respectively, were evaluated for the proposed cell configuration. For either alternative at the end of construction, the minimum friction angle (ϕ or δ) must be 20° for a FS = 1.5 for a 4H:1V slope and 26° for a 3H:1V slope (see Figure 11). For long-term stability, the affects of a phreatic surface build up above the geomembrane were evaluated yielding the following internal or interface friction angles for factor of safety of 1.5:

Slope	Slope Length	Head Buildup	Assumed Cover Soil Internal Friction Angle, ϕ_c	Internal/Interface Friction Angle for Fs= 15 (ϕ/δ)
4H:1V	120 ft.	6"	30°	22°
		12"	30°	24°
3H:1V	90 ft.	6"	30°	28°
		12"	30°	31°

See Figure 12 for typical failure surface analyzed.

For the Alternative 1 cross section which includes a PVC and Claymax liner and drainage net/filter fabric combination, the results of the analyses indicate that reinforcement of the cover soil above the liner system will be required to provide the necessary stability for the cap system. Reinforcement will be necessary since interface friction angles of filter fabric on drainage net or filter fabric on PVC may be as low as 8° to 10°[(7)]. Similarly for Claymax, the internal friction angle, typically reported to be on the order of 8°[(2)] will govern, even if the interface angle was sufficient. "Geogrids" may be used to provide the necessary reinforcing for Alternative 1 by carrying the weight of the cover soil and reducing the tensile stress in the geomembrane. Analyses have been performed to estimate the necessary strength of geogrids to effectively reduce stresses and provide adequate factors of safety against sliding. Based on analyses performed, required tension (tensile strength) of geogrids are presented in Figure 13 as a function of friction angle at the location of the least stable interface. Assuming an interface friction angle of 8°, the following required tensile strengths have been calculated for a FS of 1.5:

Cap Slope	Required Tensile Strength of Geogrid	Make/Model of Acceptable Geogrid[4]	No. of Layers of Geogrids
4H:1V	2500 plf	Tensar UX1500	1
3H:1V	4000 plf*	Tensar UX1600	2

* This value exceeds the allowable tensile strength of any one geogrid[4]. Therefore, two layers of geogrid

reinforcement are necessary to stabilize the proposed cap at a 3H:1V slope. These layers should be placed 6 inches apart directly above the upper filter fabric within the cover soil.

Alternatively, a cap consisting of a 40- or 60-mil textured HDPE and Bentomat SS liner and a geocomposite drainage layer (filter fabric bonded to the top and bottom of the HDPE drainage net) shown in Figure 2b, will eliminate the need for a geogrid reinforcement provided that the interface friction angle between all interfaces above the stabilized waste is $\geq 24°$ for a 4H:1V slope. Existing data indicates that 24° is a reasonable value to achieve[6]. The interface friction angle for a textured 40-mil HDPE liner would be identical to that for a textured 60-mil HDPE liner.

For a 3H:1V slope, utilizing Alternative 2, the minimum interface friction angle between all interfaces above the stabilized waste must be 28°. According to Daniel et al.[2], this value is not reasonable for a Bentomat Geosynthetic Clay Liner. Therefore, reinforcement of the cap will be needed for long-term stability.

RECOMMENDATIONS:

Based upon results of the analyses performed, the following recommendations are made for incorporation into the landfill closure plan and ultimately the design documents for the Gilbert and Bennett site.

Overall Containment Cell Stability

- Stability of the containment cell is attainable with properly specified requirements for material strength and placement procedures. Design documents should be developed in a manner which will provide for proper treatment and placement of waste material. Requirements should provide for a strength necessary for a factor of safety of 1.5 (i.e. $\phi \geq 26°$ for 3H:1V and $\phi \geq 21°$ for 4H:1V) under saturated and unsaturated conditions.

- To provide for adequate stability of the containment cell, the shear strength of the stabilized/solidified waste should be no less than 50 pounds per square inch (psi). With the proper crushing and placement of materials in the containment cell, the necessary strengths should be readily attained. Specifications should be developed to require stabilization/solidification and placement to specified in-situ densities which will satisfactorily yield the necessary material strengths.

- As a part of construction the Contractor should be required to perform pilot or demonstration testing prior to overall stabilization production. As a part of the pilot or demonstration testing to be performed, the Contractor should be required to provide strength test results of treated materials in order for Malcolm Pirnie to evaluate the specified placement criteria for the treated materials.

Eastern Perimeter Embankment Stability

- Based on available information, the slope of the eastern perimeter embankment will need to be regraded to a 3H:1V slope to provide a factor of safety of 1.5. Additional information is needed to assess stability if the final slope is to be steeper than 3H:1V. Concerns relate to average strength (ϕ) of in-situ embankment soils and the possible presence within the embankment of the "weaker" stratum encountered in nearby borings. Ultimately, a slope no steeper than 2.5H:1V is recommended for purposes of rip-rap placement, unless some sort of retaining (or crib-wall) type structure is provided. If a 3H: 1V slope is used, it is recommended that confirmation borings be made by the Contractor, as a first step in construction, to confirm the presence/absence of potentially weak foundation materials. If a slope steeper than 3H:1V is used, it is recommended that borings be made as part of final design.

- Traffic and equipment storage along the top of the embankment should be limited. In the text, allowable surcharge load is addressed as a function of slope geometry. Depending on the final side slope used, a final assessment of allowable surcharge should be made. Sequence of work and duration of loads are considerations which will be assessed to develop final requirements for design specifications.

Cap Stability

- If the cap is properly constructed using a texturized 60 mil HDPE and Bentomat liner and geocomposite drainage layer (Alternative 2 as addressed in text), geogrid reinforcement will not be required. It is estimated that for the 4H:1V slope and a 6" or 12" head buildup above the cap, Alternative 2 will be more cost effective than Alternative 1 (using 30 mil PVC, Claymax liner, drainage net/filter fabric combinations and a Tensar UX 1500 (or equivalent) Geogrid). Specifications should be developed which will require the Contractor to test material interfaces prior to construction of the cap. This testing should provide for:

- saturated general fill on filter fabric
- filter fabric on textured HDPE
- textured HDPE on hydrated Bentomat
- hydrated Bentomat on saturated stabilized waste

- For a 6" and 12" head buildup above the cap, both Alternative 2 cross section for a 3H:1V slope will require geogrid reinforcement. Again, specifications should require that confirmation testing be performed by the Contractor.

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